

# Comparitive Study on Cost Analysis of Framed, Grid & Truss Roofing System

## PROJECT REPORT



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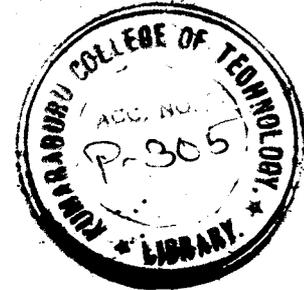
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DEPARTMENT OF CIVIL ENGINEERING

# Kumaraguru College of Technology

COIMBATORE - 641 006.

**DEPARTMENT OF CIVIL ENGINEERING**  
**Kumaraguru College of Technology**  
**COIMBATORE - 641 006.**



**CERTIFICATE**

This is the Bonafide Record of the Project Titled

**COMPARATIVE STUDY ON COST ANALYSIS OF FRAMED,  
GRID AND TRUSS ROOFING SYSTEM**

Done by Mr.

Partial fulfilment of the Degree of Bachelor of Engineering in Civil  
Engineering Branch of Bharathiar University, Coimbatore - 641 046  
during the Year 1997 - 98.

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**INTERNAL EXAMINER**

**EXTERNAL EXAMINER**

# COMPARATIVE STUDY ON COST ANALYSIS OF FRAMED, GRID AND TRUSS ROOFING SYSTEMS

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## ACKNOWLEDGEMENT

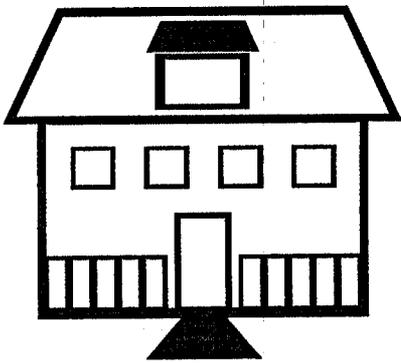
The era of Civil is a vast sea in which we have to sail for a long unreached shore. To reach the invisible shore and to complete a project with such a comprehensive and a smooth finish it is not easy to do without receiving help from various numerous sources and all other members Involved in this field.

We acknowledge a great deal of gratitude to our Professor Dr. K. SWAMINATHAN M.Tech., Ph.D., Head of the Department of Civil Engineering for his pleasant encouragement and arrangement of guidance and moral support during the course of the project.

We are greatly bound to our beloved principal Dr. S. SUBRAMANIAM. M. Tech., Ph.D., for having provided us with ample facilities to fulfil this task.

We are also proud to pay our sincere and heartfelt thanks to Mr. V.GOVINDARAJ M.E., (Structures) for his kind encouragement, valuable suggestions and his unfailing proper guidance with full pledged support at all steps of this project on behalf of getting more knowledge and new ideas that is to be established in this field which paved the way for our prosperous future life.

Last but not least we wish to express our sincere thanks and gratitude to all our faculty members of Civil Engineering Department for their generous gesture and help which has enabled us to complete the project work successfully.



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# SYNOPSIS

## SYNOPSIS

The aim of this project work is to study and compare the cost involved in different roofing systems namely

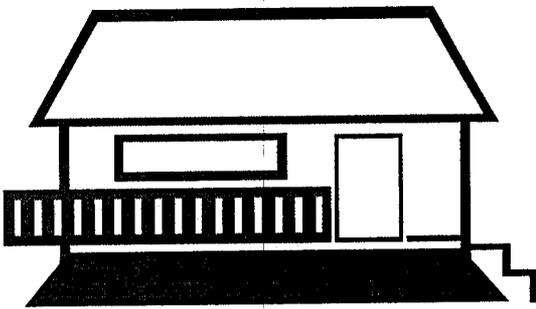
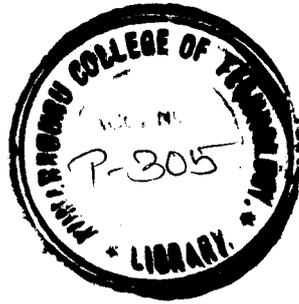
1. FRAMED STRUCTURES
2. GRID STRUCTUES
3. TRUSSES.

The study was carried out in the following steps.

1. A standard hall of length 15 metre and breadth of varying spans from 4 m to 40m at an increment of 4m is considered.
2. For each span the above three roofing systems are Analysed designed and its total construction cost estimated.
3. The results are compared and the conclusions are derived from it.

Analysis and Designing of the roofing structures were carried out partially by computer and others manually. A step by step procedure of analysis, design and estimation of the above said roofing systems and its structural drawings are also given.

The cost involved in the above said roofing systems are plotted in a graph to study the comparison of cost. An empirical formula has also been



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# INTRODUCTION

# INTRODUCTION

## 3.1 GENERAL

Though the era of construction Industry is very competitive nowadays. It is in the hands of the Engineers to give an durable project to their clients to hold on in this valuable field. It is the part of an effective engineer to give an “DURABLE STRUCTURE ALONG WITH QUALITY AND ECONOMY”. For this the Engineer should select the proper foundations, superstructure and other aspects.

In this project, an attempt has been made to study the cost of three different Roofing systems, i.e., Frames, Trusses & Grids which are now vastly used in construction nowadays. A standard hall of length of 15m and breadth varying from 4m to 40m are chosen for our study on cost comparison.

For proper understanding and easy selection, of the particular roofing system the comparison is plotted in graph. Various other conclusions are also made in this project.

The analysis and design at these structures were carried out partially by computer and manually. An existing computer program developed by the department is used for the analysis and design purpose.

Computer Analysis was carried out for Frames & Trusses Rather the Analysis and design of Grid structures and the design of Frames & Trusses were carried out manually.

However Estimation, both Detailed and Abstract was carried out manually, proper steps has been indicated in designing & Estimations carried out in this project.

### **3.2 REVIEW OF LITERATURE**

Before proceeding the project a clear study was made with the previous Literature available regarding the works available.

A detailed review was made on the project titled comparative study of Roofing systems done in the year 1993-94 guided by Mr. NAGABUSHNA B.E., M. Tech., In this project work a comparative study on three roofing system i.c., Continuous slab, Grid beam slab & Folded plate roofing were carried out. This project work was any how carried out to only a standard size of hall 12m x 40m and the aim was to get a column free space. The designs were carried out by limit state method.

The conclusion of the above project were as follows.

1. There is no difference in cost between continuous slab and grid beam slab but the cost of folded plate roofing was 18.6% higher.

2. Suggestions was given to adopt Grid slab roofing, taking in consideration of Economy and good appearance after construction. Any how if cost criteria is not considered folded plates can be adopted for its better look for a big hall when compared to the other roofing systems.

The aim at our project varies from this as given in synopsis many details are taken in consideration from this project.

# **PRINCIPLES OF ANALYSIS AND DESIGN**

As mentioned in the Introduction of this project there are three types of structures considered namely Trusses, Grids & Frames, the height of the roofing structure is taken commonly as 5m and the size of the column varied depending upon the span. The various symbols used in analysis and design are mentioned below. the principles and the method adopted are also listed below. However only an example of the Analysis and design are listed in Detail other results are shown in tabulation.

## **FRAMED STRUCTURES**

The Analysis of the framed structures was carried out by the available program written in Fortran 77 which is based on stiffness method. The output of shear force and Bending moment was obtained for the input data consisting of various details of Frame i.e., No of members in the Frame, No of nodes, Loads, Modulus of elasticity, moment of inertia, and the length of the members.

The design of these structures was carried out by Limit state method. For which proposed loading was attained from the appropriate code books. The design was carried out in steps as per the Explanatory code book of Is-456 i.e., SP-16.

The Grade of concrete used was m15 & Steel Fe 415 at characteristic strength  $15\text{N/mm}^2$  of  $415\text{N/mm}^2$  respectively. The following steps were followed During the design of framed structures.

1. Bending moment & shear force (By computer analysis)
2. Necessary data assumption.
3. Design of beam
  - a. Mid span section.
  - b. Support section.
4. Check for shear.
5. Design of two way continuous slab.

## **GRID STRUCTURES**

The complete analysis and design of the Grid structures was carried out by using plate theory. The design was carried by limit state method and concrete of grade m15 of steel of Grade Fe415 was used. The size of the grid beam was taken as 230 x 600mm with a panel span of 2m longitudinally and 3m laterally. The panels vary from 10 to 100 at an increment of 10 for each condition, i.e., span from 4m to 40m with an increment of 4m.

Proper use of SP-16 was carried out for the design of shear and Torsion. Dead & live loads also taken from the appropriate code books.

The steps carried out in Analysis and design are as follows.

1. Assuming Appropriate datas.
2. Calculation of Loads.
3. Calculation of deflection equation.
  - a. Bending moment equation.
  - b. Shear force equation.
  - c. Torsion equation.
4. Calculation of maximum Bending moment.
5. Calculation of shear force
6. Calculation of Torsion.
7. Design of shear and torsion Reinforcement.

## TRUSS ANALYSIS

The analysis of Trusses was carried out by the available program written in Fortran 77 which was programmed using stiffness method. The output of axial force was obtained for the given Input datas consisting of No of members in the frames, No of Nodes, Loads, Modulus of Elasticity and the length of the members.

The design of these members was carried out by working stress method the loading was obtained from the necessary code books. The design was carried out by as per IS 800 and proper use of it has been made.

Angle section has been used back to back for both bottom chord and top chord members and single section has been used for intermediate member. The connection of member was done by Gusset plates with riveted connections. The following steps was carried out in the design of frames.

1. Axial force (By computer analysis)
2. Design of Top Chord member
3. Design of Bottom Chord member
4. Design of Intermediate member
5. Design of Joints.

An example of each structure is illustrated along with its analysis, Design Estimations, and Structural drawings in the fourth coming chapters.

## SYMBOLS USED

$f_{ck}$	-	Characteristic Compressive strength of concrete
A	-	Area
D	-	Total Depth
d	-	Effective depth
b	-	Breadth
$f_y$	-	Characteristic strength of steel
$l_x$	-	length of shorter side of span
$l_y$	-	length of longer side of span.
BM	-	Bending moment.
$\sigma_{cbc}$	-	Permissible stress in concrete in bending compression
$\sigma_{st}$	-	Permissible stress in steel in tension
$\tau_c$	-	Shear stress in concrete
$\tau_v$	-	Nominal shear stress
$\phi$	-	Diameter of Bar

- $\tau_{cmax}$  - Max. shear stress in concrete with shear reinforcement
- Q - Intensity of load
- E - Modulus of Elasticity of concrete
- $\mu$  - Poission ratio
- GJ - Modulus of Rigidity.
- I - Moment of Inertia.
- $V_e$  - equivalent shear
- $\tau_{ve}$  - equivalent shear stress

## 4.2. Design of Framed Structure

Data assumed

Span = 4m

Concrete Grade M15,

Steel Grade Fe415

### a. Design at support

assume 16mm  $\phi$  bars are used

effective cover = 25+8 = 33mm

Ultimate moment = 1.5 x 22.49 (From out File)

$$= 33.74 \text{ KN}$$

$$\begin{aligned} \text{Mul i m} &= 0.36 \times \frac{X_{\text{urax}}}{d} \left[ 1 - 0.42 \times \frac{X_{\text{urax}}}{d} \right] f_{ck} b d^2 \\ &= 0.36 \times 0.48 [1 - 0.42 \times 0.48] 15 \times 300 \times 267^2 \\ &= 38.72 \text{ KN} > 33.74 \end{aligned}$$

Hence the section is singly reinforced

$$\text{Ast required} = 400.5 \text{ mm}^2$$

Provide 12mm  $\phi$  bars of 4 Nos

$$\text{Provided steel area} = 452 \text{ mm}^2$$

## b. Design at midspan

Maximum +ve bending moment

$$= 78 - 33.74$$

$$= 44.26 \text{ KN.m}$$

$$\text{Ast required} = 547 \text{ mm}^2$$

Alternate bars are bent upto midspan, so support area by 2 is continued to midspan.

$$\text{Net Ast Req.} = 3.51 \text{ mm}^2$$

Provide 4 Nos of 12 mm  $\phi$  bars

$$\text{Provide steel area} = 452 \text{ mm}^2$$

## c. Design of Shear

$$\text{Nominal Shear stress } \tau_v = \frac{V_u}{bd}$$

$$= \frac{40 \times 10^3}{300 \times 267}$$

$$= 0.05 \text{ N/mm}^2$$

$$\tau_{c\text{max}} = 2.5 \text{ N/mm}^2 \text{ (From IS 456)}$$

$$> \tau_v$$

Percentage of tensile reinforcement = 0.5% (From IS 456)

$$\tau_c = 0.46 > \tau_v$$

$$< \tau_{c\text{max}}$$

Since  $\tau_v < \tau_c$  the minimum reinforcement shall be provided in the form of stirrups (From IS456 Art 25.5.1.6)

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{f_y}$$

Assume 6 mm  $\phi$  bars of 2 legged

$$\therefore \text{Stirrups Spacing } S_v = \frac{A_{sv} \times f_y}{0.4 \times b}$$

$$\text{Here } A_{sv} = 56.5 \text{ mm}^2$$

$$b = 300 \text{ mm}$$

$$f_y = 415 \text{ N/mm}^2$$

Provide 6mm  $\phi$  bars @ 190 mm c/c

#### DESIGN OF TWO WAY CONTINUOUS SLAB :

##### a. DATA :

$$\text{Shorter side dimension} = 3 \text{ m}$$

$$\text{Longer side dimension} = 4 \text{ m}$$

$$\text{Total length} = 15 \text{ m}$$

$$\text{Side ratio } l_y / l_x = 1.33$$

$$\text{approx. Span depth} = 200 \text{ mm}$$

$$\text{Effective depth} = 200 - (15+5)$$

$$= 180 \text{ mm}$$

$$\text{Effective Span} = 3 \text{ m}$$

## b. LOAD CALCULATION

$$\text{Dead Load} = 0.2 \times 25 = 5 \text{ KN/m}^2$$

$$\text{Roof Finish} = 1 \text{ KN/m}^2$$

$$\text{Live load} = 3 \text{ KN/m}^2$$

	$\alpha_x$	$\alpha_y$
Two edges discontinuous		
one edge continuous (-ve)	0.77	-
(+ve)	0.58	0.043
Intermediate, two edge		
continuous two short edge		
discontinuous (-ve)	0.057	-
(+ve)	0.044	0.035

$$M_x = \alpha_x Wl_x^2$$

$$M_y = \alpha_y Wl_y^2$$

Max. BM is x direction

$$M_x = 2.77 \text{ KN-m}$$

Max. BM is y direction

$$M_y = 1.55 \text{ KN}$$

$$1.5 M_x = 4.16 \text{ KN-m}$$

$$1.5 M_y = 2.33 \text{ KN-m}$$

Provide 8 mm  $\phi$  4 bars at 200 c/c in the both the direction (From SP 16 table 22)

**c. Estimation of Frame**

Span = 4 m

$$\begin{aligned}\text{Qty. of concrete in slab} &= 0.2 \times 4 \times 15 \\ &= 12\text{m}^3\end{aligned}$$

$$\begin{aligned}\text{Qty. of concrete in beam} &= 0.25 \times 0.3 \times 4 \times 6 \\ &= 1.8 \text{ m}^3\end{aligned}$$

$$\begin{aligned}\text{Total} &= 12 + 1.8 \\ &= 13.8\text{m}^3\end{aligned}$$

$$\begin{aligned}\text{Steel for slab} &= 20 \times \frac{\pi}{4} \times 0.008^2 \times 7850 \times 75 \times \frac{\pi}{4} \times 0.008^2 \times 7850 \\ &= 237 \text{ Kg}\end{aligned}$$

$$\begin{aligned}\text{Steel for beam} &= 4 \times \frac{\pi}{4} \times 0.012^2 \times 7850 \times 6 \\ &\quad + 4 \times \frac{\pi}{4} \times 0.012^2 \times 7850 \times 6 \\ &\quad + 1.5 \times 0.006^2 \times \frac{\pi}{4} \times 7850 \times 138 \\ &= 222 \text{ Kg}\end{aligned}$$

$$\text{Total Steel} = 459 \text{ Kg}$$

$$\text{Cost of concrete/m}^3 = \text{Rs. } 3250$$

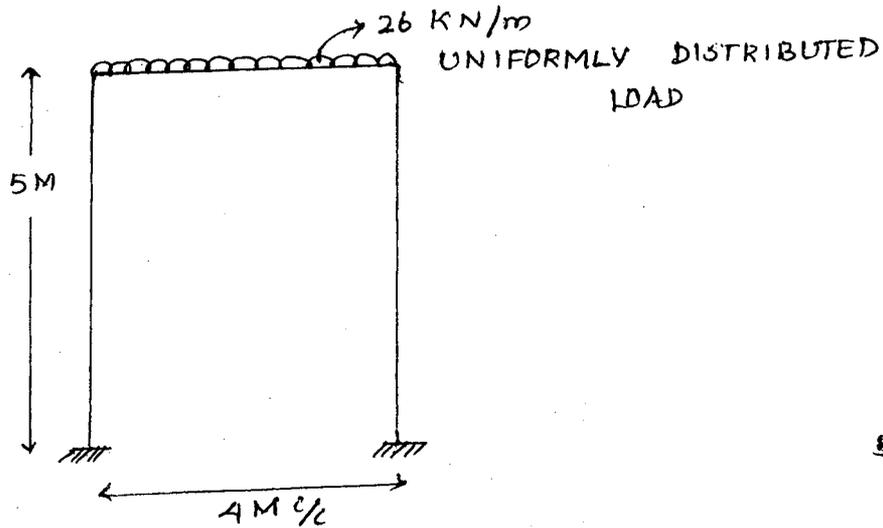
$$\text{Cost of steel/Kg} = \text{Rs. } 16$$

$$\begin{aligned}\text{Cost for total Qty. of concrete} &= 3250 \times 13.8 \\ &= \text{Rs. } 41340\end{aligned}$$

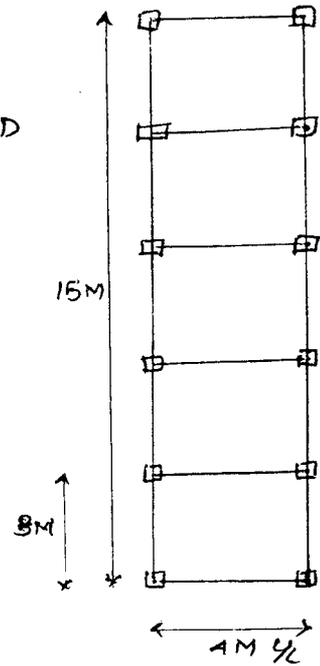
$$\begin{aligned}\text{Cost for total Qty. of steel} &= 16 \times 459 \\ &= \text{Rs. } 7344\end{aligned}$$

$$\text{Total cost} = \text{Rs. } 48684$$

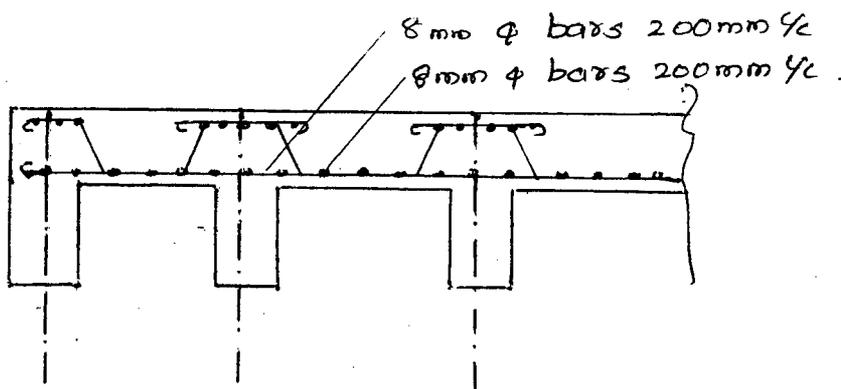
# DETAILING OF FRAME



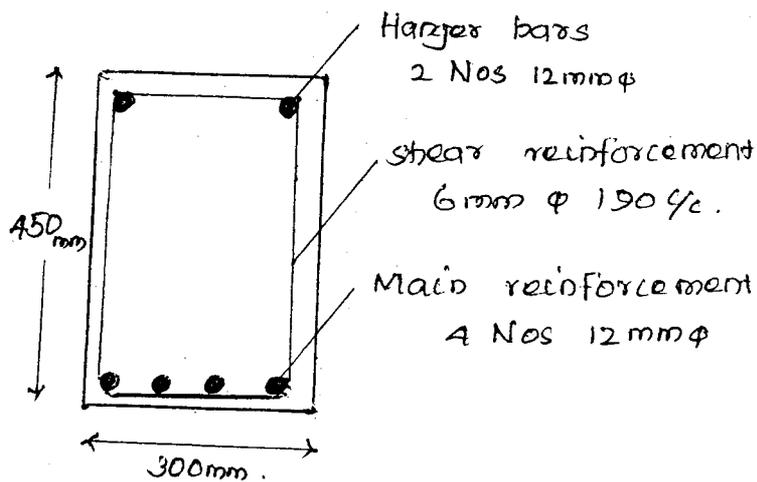
LOADING OF FRAME



PLAN OF HALL



LONGITUDINAL CROSS SECTION



CROSS SECTION OF BEAM

FRAME

SPAN- ----4METRE

INPUT DATA

4 3 6 22.076e6

1 0 0

2 0 5

3 4 5

4 4 0

1 1 2 .135 2.278e-3

2 2 3 .09 .675e-3

3 3 4 .135 2.278e-3

0

1

2 0 -40 26.67 0 -40 -26.67

1 0

2 0

3 0

10 0

11 0

12 0

FRAME

SPAN ----- 0.4METRE

MEMBER FORCES IN LOCAL COORDINATE SYSTEM

MEMBER	NODE	AXIALFORCE	SHEARFORCE	BENDINGMOMENT
1	1	0.400000E+02	-0.673876E+01	0.112040E+02
	2	-0.400000E+02	0.673876E+01	0.224898E+02
2	2	0.673876E+01	0.400000E+02	-0.224898E+02
	3	-0.673876E+01	0.400000E+02	0.224898E+02
3	3	0.400000E+02	0.673876E+01	-0.224898E+02
	4	-0.400000E+02	-0.673876E+01	-0.112040E+02

# DESIGN OF FRAMES

SHEAR DESIGN																
DESIGN AT MID SPAN					DESIGN AT SUPPORT											
SPAN	FACTORED (MU) MOMENT	Mu lim (KN-m)	Pt	Ast1 (mm <sup>2</sup> )	Net BM Wt/ 8- Mu =Me1	Mu/ bd <sup>2</sup>	Pt	Pc	Net Ast (Ast-1/2 Ast1) (mm <sup>2</sup> )	Asc	$\tau_v = v_u / bd$ (N/mm <sup>2</sup> )	$\tau_c$	$V_{us} = V_u - \tau_c bd$	$S_v = 0.87fy \frac{A_{sv}}{V_{us}}$	Ast Provided	REMARKS
4m	33.74	0.65 (D=450 mm)	0.19	400 4 bars of 12mm $\phi$	44.26	0.85	0.256	-	351 4bars of 12mm $\phi$	(Mulin>Mu) So singly reinforced section	0.05	0.46	-	190	6mm $\phi$ @ 190mm c/c	$\tau_v < \tau_c$ so min Reinforcement is provided.
8m	58.44	0.9 (D=500 mm)	0.270	457 6 bars of 12mm $\phi$	150	2.4	0.64	-	897 4bars of 16mm $\phi$	-	0.67	0.5	23874	225	2 legged 8mm $\phi$ @ 200mm c/c	Mulin>Mu so singly reinforcement
12m	276	1.7 (D=600 mm)	0.558	937 6 bars of 16mm $\phi$	303	2.8	0.48	0.9	1104 4 bars of 20mm $\phi$	1850 4 bars of 20mm $\phi$	0.70	0.5	33974	214	2 legged 8mm $\phi$ @ 200mm c/c	
16m	365	1.82 (D=700 mm)	0.602	1170 6 bars of 16mm $\phi$	655	2.91	2.4	1.1	1884 6 bars of 20mm $\phi$	1884 6 bars of 20mm $\phi$	0.70	0.5	42950	193	2 legged 8mm $\phi$ @ 180mm c/c	
20m	467	1.9 (D=900 mm)	0.641	1292 6 bars of 18mm $\phi$	1039	3.11	1.5	1.2	2280 6 bars of 22mm $\phi$	2513 8 bars of 20mm $\phi$	0.77	0.5	94950	191	2 legged 8mm $\phi$ @ 180mm c/c	
24m	481	1.95 (D=1000 mm)	0.667	1700 6 bars of 20mm $\phi$	1390	3.82	1.6	1.3	3041 6 bars of 22mm $\phi$	3141 10 bars of 20mm $\phi$	0.83	0.5	95960	183	2 legged 8mm $\phi$ @ 180mm c/c	
28m	506	1.97 (D=1200 mm)	0.672	2180 6 bars of 22mm $\phi$	2042	4.41	1.98	1.31	3801 10 bars of 22mm $\phi$	3770 12 bars of 20mm $\phi$	0.84	0.51	114915	167	2 legged 8mm $\phi$ @ 160mm c/c	
32m	926	2.1 (D=1400 mm)	0.712	2800 8 bars of 22mm $\phi$	2402	4.7	1.99	1.35	3950 8 bars of 25mm $\phi$	3801 10 bars of 22mm $\phi$	0.89	0.51	135955	165	2 legged 8mm $\phi$ @ 160mm c/c	
36m	975	2.3 (D=1500 mm)	0.713	2965 8 bars of 22mm $\phi$	3237	4.95	2.1	1.37	4110 8 bars of 25mm $\phi$	4562 12 bars of 22mm $\phi$	0.9	0.51	139950	145	2 legged 8mm $\phi$ @ 140mm c/c	
40m	1445	2.32 (d=1600 mm)	0.712	3150 10 bars of 22mm $\phi$	3755	5.29	2.3	1.49	4220 10 bars of 25mm $\phi$	4980 10 bars of 25mm $\phi$	0.93	0.51	160250	141	2 legged 8mm $\phi$ @ 140mm c/c	

## ESTIMATION OF FRAMES

SPAN	ESTIMATION OF CONCRETE				ESTIMATION OF STEEL				TOTAL COST (Rs.)
	SLAB (m <sup>3</sup> )	BEAM (m <sup>3</sup> )	TOTAL (m <sup>3</sup> )	RATE (Rs.)	SLAB (kg)	BEAM (kg)	TOTAL (kg)	RATE (Rs.)	
	4m	12	0.72	12.72	41,340	237	222	459	
8m	24	4.32	28.32	92,040	503	444	947	15,153	1,07,193
12m	36	8.64	44.64	1,45,080	755	993	1,747	27,966	1,73,046
16m	48	20.16	68.16	2,21,520	1,006	5,481	6,487	1,03,790	3,25,310
20m	60	28.50	88.20	2,76,900	1,258	6,416	7,674	1,22,780	3,99,680
24m	72	43.20	115.20	3,74,400	1,510	9,830	11,340	1,81,438	5,55,838
28m	84	60.48	144.48	4,69,560	1,762	17,965	19,725	3,15,607	7,85,167
32m	96	80.64	176.64	5,74,080	2,014	18,826	20,840	3,33,446	9,07,526
36m	108	97.20	205.20	6,66,900	2,266	28,808	31,074	4,97,181	11,64,081
40m	120	115.20	235.20	7,64,400	2,518	32,103	34,621	5,53,942	13,18,342

## Design of Grid Structure

### DATA ASSUMED

$$\text{Span} = 4\text{m}$$

$$\text{Size of beam} = 230 \times 600 \text{ mm}$$

$$\text{Thickness of slab} = 120\text{mm}$$

#### a. Load Calculation

$$\begin{aligned}\text{Self Wt of Slab} &= 15 \times 4 \times 0.12 \times 25 \\ &= 180 \text{ KN}\end{aligned}$$

$$\begin{aligned}\text{Self Wt of beam in X-direction} &= 6 \times 0.23 \times 0.48 \times 25 \times 4 \\ &= 66.24 \text{ KN}\end{aligned}$$

$$\begin{aligned}\text{Self Wt of beam in Y- direction} &= 3 \times 0.23 \times 0.48 \times 25 \times 15 \\ &= 41.4 \text{ KN}\end{aligned}$$

$$\text{Floor Finish} = 1 \text{ KN/m}^2$$

$$\text{Which gives } 1 \times 4 \times 15 = 60 \text{ KN}$$

$$\text{Live Load} = 1.5 \text{ KN/m}^2$$

$$\Rightarrow 1.5 \times 4 \times 15 = 90 \text{ KN}$$

$$\text{Total Load} = 427 \text{ KN}$$

$$\text{Intensity of load} = \frac{427}{4 \times 15} = 7.3 \text{ KN / m}^2$$

## b. Calculation of deflection equation

The vertical deflection at any point of grid

$$a = \frac{16q}{\pi^6} \left[ \frac{\sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)}{\frac{D_x}{a_x^4} + \frac{2H}{a_x^2 b_y^2} + \frac{D_y}{b_y^4}} \right]$$

Here,  $a_x = 4 \text{ m}$

$$b_y = 15 \text{ m}$$

$$D_x = \frac{EI_1}{b_1} ; D_y = \frac{EI_2}{a_1}$$

$$E = 5700\sqrt{f_{ck}} = 5700\sqrt{15} = 22076 \times 10^6 \text{ KN / m}^2$$

$$I_1 = I_2 = \frac{bd^3}{12} = \frac{0.23 \times 0.6^3}{12} = 4.14 \times 10^{-3} \text{ m}^4$$

$$a_1 = 2 \text{ m}$$

$$b_1 = 3 \text{ m}$$

$$D_x = \frac{22076 \times 10^6 \times 4.14 \times 10^{-3}}{3} = 30465$$

$$D_y = \frac{22076 \times 10^6 \times 4.14 \times 10^{-3}}{2} = 45697$$

$$2H = C_x + C_y$$

$$C_x = \frac{C_1}{b_1} ; C_y = \frac{C_2}{a_1}$$

$$C_1 = C_2 = GJ = \frac{E}{2(1+\mu)} \left[ hb^3 \left[ \frac{1}{3} - 0.21 \frac{b}{h} \left( 1 - \frac{b^4}{12h^4} \right) \right] \right]$$

$$= \frac{22.076 \times 10^6}{(1+0.25)^2} \left[ 0.6 \times 0.23^3 \left[ \frac{1}{3} - 0.21 \times \frac{0.23}{0.6} \left( 1 - \frac{0.23^4}{12 \times 0.6^4} \right) \right] \right]$$

$$= 16308$$

$$2H = \frac{c_1}{b_1} + \frac{c_2}{a_7} = 13590$$

$$\therefore a = \frac{16 \times 7.3}{\pi^6} \left[ \frac{\sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)}{\frac{30465}{4^4} + \frac{13590}{4^2 \times 15^2} + \frac{45697}{15^4}} \right]$$

$$a = 0.00098 \sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)$$

$$\frac{\partial a}{\partial x} = 0.00077 \cos\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)$$

$$\frac{\partial^2 a}{\partial x^2} = -0.0006 \sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)$$

$$\frac{\partial^2 a}{\partial x^2} = -0.000043 \sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)$$

$$\frac{\partial^2 a}{\partial x \partial y} = 0.00016 \cos\left(\frac{\pi x}{ax}\right) \cos\left(\frac{\pi y}{by}\right)$$

**c. To find BM equation**

$$\text{BM in x - direction } M_x = -D_x \left( \frac{\partial^2 a}{\partial x^2} \right)$$

$$\text{BM in y - direction } M_y = -D_y \left( \frac{\partial^2 a}{\partial y^2} \right)$$

$$M_x = -30465 (-0.0006) \sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)$$

$M_x = 18.28 \sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)$
$M_x = 1.96 \sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right)$

**d. To find Torsion eqn**

Torsional moment about xy axis:

$$M_{xy} = -\frac{c_1}{b_1} \left( \frac{\partial^2 a}{\partial x \partial y} \right)$$

$$M_{yx} = -\frac{c_2}{a_1} \left( \frac{\partial^2 a}{\partial x \partial y} \right)$$

$M_{xy} = -0.87 \cos\left(\frac{\pi x}{ax}\right) \cos\left(\frac{\pi y}{by}\right)$
$M_x = 1.96 \cos\left(\frac{\pi x}{ax}\right) \cos\left(\frac{\pi y}{by}\right)$

**e. To find shear Force eqn**

The shear Force along X-direction

$$Q_x = \frac{-\partial}{\partial x} \left[ D_x \left( \frac{\partial^2 a}{\partial x^2} \right) + \frac{c_2}{a_1} \left( \frac{\partial^2 a}{\partial x \partial y} \right) \right]$$

The shear Force along Y-direction

$$Q_y = \frac{-\partial}{\partial y} \left[ D_y \left( \frac{\partial^2 a}{\partial y^2} \right) + \frac{c_2}{a_1} \left( \frac{\partial^2 a}{\partial x \partial y} \right) \right]$$

$$\Rightarrow Q_x = \frac{-\partial}{\partial x} \left[ -18.28 \sin\left(\frac{\pi x}{ax}\right) \sin\left(\frac{\pi y}{by}\right) + 0.87 \cos\left(\frac{\pi x}{ax}\right) \cos\left(\frac{\pi y}{by}\right) \right]$$

$$= -18.28 \left( \frac{\pi}{ax} \right) \cos \left( \frac{\pi x}{ax} \right) \sin \left( \frac{\pi y}{by} \right) + 0.87 \left( \frac{\pi}{ax} \right) \sin \left( \frac{\pi x}{ax} \right) \cos \left( \frac{\pi y}{by} \right)$$

$Q_x = 14.36 \cos \left( \frac{\pi x}{ax} \right) \sin \left( \frac{\pi y}{by} \right) + 1.02 \sin \left( \frac{\pi x}{ax} \right) \cos \left( \frac{\pi y}{by} \right)$
$Q_y = 0.41 \sin \left( \frac{\pi x}{ax} \right) \cos \left( \frac{\pi y}{by} \right) + 0.18 \cos \left( \frac{\pi x}{ax} \right) \sin \left( \frac{\pi y}{by} \right)$

The calculation of Maximum bending moment, Torsion and Shear

Forces are shown in Table.

## f. Design of Shear and Torsion

### I. Shear Reinforcement

DATA

Tensile Reinforcement = 0.8%

Beam Size = 0.23 × 0.6m

Concrete Grade = M15

Steel Grade = Fe415

Factored Shear Force = 1.5 × 13.66 = 20.49KN

Effective depth d = 600 - 35 = 565 mm

$$\begin{aligned} \text{Nominal Shear Stress} = \tau_v &= \frac{V_u}{bd} \\ &= \frac{20.49 \times 10^3}{230 \times 565} \\ &= 0.16 \text{ N / mm}^2 \end{aligned}$$

From SP16 Table - J for M15

$$\tau_{v_{\max}} = 2.5 \text{ N / mm}^2$$

$$\tau_v < \tau_{\text{cmax}}$$

Hence the section is not to be redesign.

From Table - 61 For  $p_t = 0.8\%$

$$\tau_c = 0.5 \text{ N / mm}^2$$

$$\tau_v < \tau_c$$

Therefore provide minimum reinforcement of 6mm  $\phi$  of 2 legged stirrups @ 190mm c/c. (including bending moment)

## II . Torsion Reinforcement

### i. Design of flexural reinforcement

From table

$$\text{Factored torsional moment} = 1.5 \times 1.3 = 1.95 \text{ KN-m}$$

$$\text{Factored shear force} = 1.5 \times 13.66 = 20.49 \text{ KN}$$

$$\text{Factored bending moment} = 1.5 \times 17.39 = 26.09 \text{ KN-m}$$

From SP-16 Page No 175

$$\begin{aligned} \text{Equivalent Shear } V_e &= V + \frac{1.6 T}{b} \\ &= 20.49 + \frac{1.6 \times 1.95}{0.23} \\ &= 34.06 \text{ KN} \end{aligned}$$

$$\begin{aligned} \text{Equivalent Shear Stress } \tau_{ve} &= \frac{V_e}{bd} \\ &= \frac{34.06 \times 10^3}{23 \times 56.5 \times 10^3} \\ &= 0.16 \text{ N / mm}^2 \end{aligned}$$

From Table - J  $\tau_{cmax} = 2.5 \text{ N/mm}^2$

For Tensile reinforcement = 0.8%,  $\tau_c = 0.55 \text{ N/mm}^2$

$$\tau_c < \tau_{ve}$$

Therefore equivalent bending moment

$$\begin{aligned} M_{e1} &= M_u + M_t \\ &= M_u + \frac{T_u (1 + \frac{D}{b})}{1.7} \\ &= 26.09 + \frac{1.95 (1 + \frac{0.6}{0.23})}{1.7} \\ &= 30.23 \text{ KN - m} \end{aligned}$$

$$\frac{M_{e1}}{bd^2} = 0.42$$

From SP-16 Table -1 For  $f_y = 415$ ,  $f_{ck} = 15$

$$P_t = 0.114$$

$$A_{st} = 148 \text{ mm}^2$$

Provide 10mm  $\phi$  of 2 bars.

## ii. Design of Side Reinforcement

Since the depth of beam is more than 450mm, Side face reinforcement of 0.05% on each side is to be provided as per IS456 Art

### 25.1.1.7

$$\begin{aligned} \text{Area required for each bar} &= 0.05 \times 23 \times 28.25 \times 100 \\ &= 32.48 \text{ mm}^2 \end{aligned}$$

Provide 12mm  $\phi$  of 1 bar on each side.

### iii. Design of Transverse Reinforcement

$$\begin{aligned}\text{Area of 2 legs of stirrups } A_{sa} &= (\tau_{ve} - \tau_c)b \\ &= 15 \text{ N/mm}\end{aligned}$$

Here, the area is very minimum Hence provide 6 mm  $\phi$  of 2 legged stirrups @ 190mm c/c.

### g. Design of Two Way Slab

DATA :

$$\text{Shorterside dimension} = 2\text{m}$$

$$\text{Longerside dimension} = 3\text{m}$$

$$\text{Side Ratio} = 3/2$$

$$= 1/5 < 2$$

The Section is to be designed as two way slab.

From IS-456, Page 138, table 20

$$\text{Short Span Co-efficient } \alpha_x = 0.075$$

$$\text{Long Span Co-efficient } \alpha_y = 0.047$$

### LOAD CALCULATION:

$$\text{Dead load of slab} = 25 \times 0.12$$

$$= 3\text{KN/m}^2$$

$$\text{Floor Finish} = 1\text{KN/m}^2$$

$$\text{Live load} = 3\text{KN/m}^2$$

$$\text{Total load} = 7\text{KN/m}^2$$

$$\begin{aligned} \text{BM in X-Direction } M_x &= \alpha_x WLx^2 \\ &= 0.075 \times 7 \times 2^2 \\ &= 2.1 \text{ KN-m} \end{aligned}$$

$$\begin{aligned} \text{BM in Y-Direction } M_y &= \alpha_y WLx^2 \\ &= 0.047 \times 7 \times 2^2 \\ &= 1.32 \text{ KN-m} \end{aligned}$$

Since the BM is minimum, therefore provide minimum reinforcement of 8mm  $\phi$  @ 200 mm c/c.

#### **h. Estimation of Grid Structure**

Span	=40 m
Size of beam in X-Direction	= 230mm $\times$ 600mm
Size of beam in Y-Direction	= 230mm $\times$ 600mm
Thickness of slab	= 120mm

#### **i. Reinforcement used:**

For Slab:

8mm 4 @ 200mm c/c in both the direction

For beam;

6mm  $\phi$  of 2 legged stirrups @ 190mm c/c as shear reinforcement

10 mm  $\phi$  of 2 bars as flextural reinforcement

12 mm  $\phi$  of 2 bars as side reinforcement

6 mm  $\phi$  of 2 legged stirrups @ 190mm c/c as transverse reinforcement

$$\begin{aligned}\text{Qty. of concrete in slab} &= 0.12 \times 15 \times 4 \\ &= 7.2\text{m}^3\end{aligned}$$

$$\begin{aligned}\text{Qty. of concrete in beam} &= 0.23 \times 0.48 \times 4 \times 6 + 15 \times 3 \times 0.23 \times 0.48 \\ &= 7.62\text{m}^3\end{aligned}$$

$$\text{Total concrete quantity} = 14.82\text{m}^3$$

$$\begin{aligned}\text{Steel in slab} &= 75 \times \frac{\pi}{4} \times 0.008^2 \times 7850 \times 4 + \\ &\quad 20 \times \frac{\pi}{4} \times 0.008^2 \times 7850 \times 15 \\ &= 236 \text{ Kg}\end{aligned}$$

ii. Steel in beam:

$$\begin{aligned}\text{Shear reinforcement} &= 363 \times 1.66 \times \frac{\pi}{4} \times 0.006^2 \times 7850 \\ &= 133\text{Kg}\end{aligned}$$

$$\begin{aligned}\text{Flexural reinforcement} &= 2 \times 6 \times 4 \times \frac{\pi}{4} \times 0.010^2 \times 7850 \\ &\quad + 3 \times 2 \times 15 \times \frac{\pi}{4} \times 0.01^2 \times 7850 \\ &= 55\text{Kg}\end{aligned}$$

$$\begin{aligned}\text{Side reinforcement} &= 2 \times 6 \times 4 \times \frac{\pi}{4} \times 0.012^2 \times 7850 \\ &\quad + 3 \times 2 \times 15 \times \frac{\pi}{4} \times 0.01^2 \times 7850 \\ &= 42\text{Kg}\end{aligned}$$

$$\begin{aligned}\text{Transverse reinforcement} &= 363 \times 1.66 \times \frac{\pi}{4} \times 0.006^2 \times 7850 \\ &= 28\text{Kg}\end{aligned}$$

Cost for concrete in Grid =  $3250 \times 14.82$

= Rs.48165

Cost for steel in Grid =  $16 \times 494$

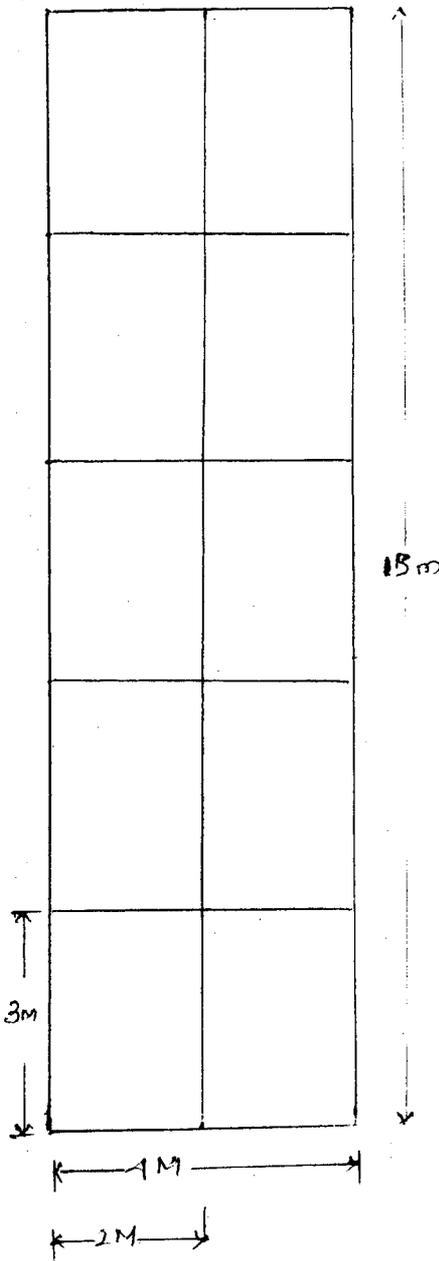
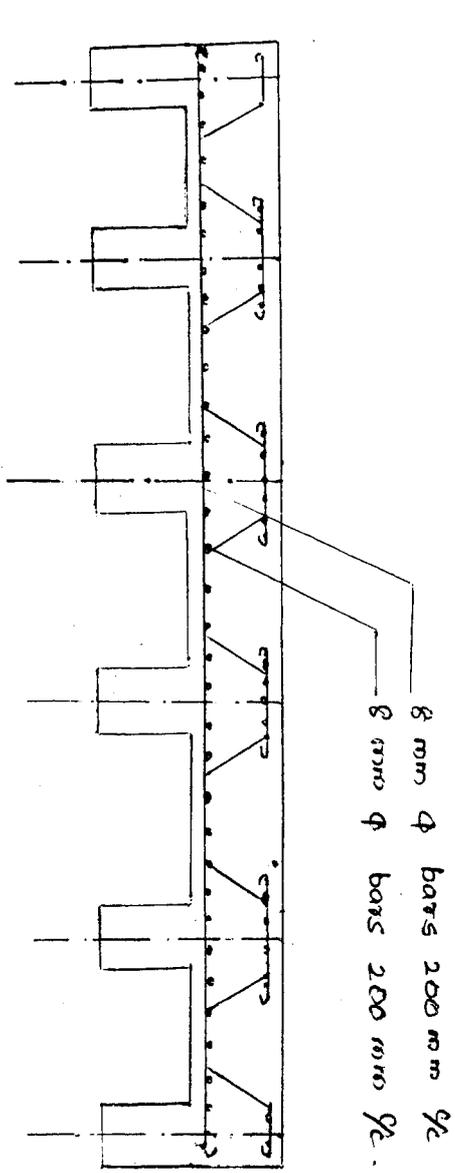
= Rs. 7915

Total Cost =  $48165 + 7915$

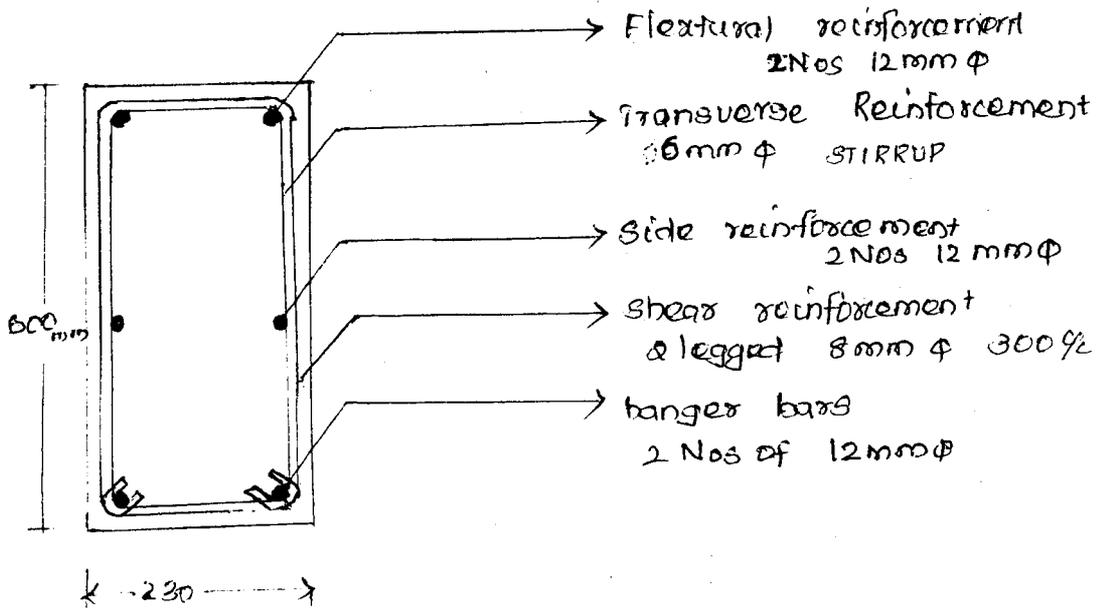
= Rs. 56080

# DETAILING OF GRID ROOF

LONGITUDINAL CROSS SECTION



PLAN OF HALL



# ANALYSIS OF GRID

Span	Mx	My	Mxy	Myx	Qx	Qy
C1	18.28 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	1.96 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	-0.87 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$	1.3 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$	14.36 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$ + 1.02 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$	0.41 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$ + 0.18 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$
C2	60.93 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	27.42 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	-6.52 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$	-9.78 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$	23.93 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$ + 3.84 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$	5.74 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$ + 1.37 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$
C3	97.49 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	91.39 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	-16.31 $\frac{5x}{ax} \frac{5y}{by} \cos$	-24.46 $\frac{5x}{ax} \frac{5y}{by} \cos$	25.52 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$ + 6.4 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$	19.14 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$ + 3.42 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$
C4	94.44 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	164.5 $\frac{5x}{ax} \frac{5y}{by} \sin$	-17.94 $\frac{5x}{ax} \frac{5y}{by} \cos$	-26.91 $\frac{5x}{ax} \frac{5y}{by} \cos$	18.54 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$ + 5.28 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$	34.45 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$ + 3.76 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$
C5	79.21 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$	210.21 $\frac{5x}{ax} \frac{5y}{by} \sin$	-19.03 $\frac{5x}{ax} \frac{5y}{by} \cos$	-28.54 $\frac{5x}{ax} \frac{5y}{by} \cos$	12.44 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$ + 4.48 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin \cos$	44.03 $\frac{\pi x}{ax} \frac{\pi y}{by} \cos$ + 3.99 $\frac{\pi x}{ax} \frac{\pi y}{by} \sin$



MAX. BM, MAX. SF & MAX. TORSION							
	CO	Mx	My	Mxy	Myx	Qx	Qy
C A S E I	0,0	0	0	-0.87	-1.3	0	0
	0,6	0	0	-0.27	-0.4	13.66	0.17
	2,6	17.39	1.86	0	0	0.32	0.13
C A S E II	0,0	0	0	-6.52	-9.78	0	0
	0,6	0	0	-2.01	-3.02	22.76	1.3
	2,6	57.95	26.08	0	0	1.19	1.77
C A S E III	0,0	0	0	-16.31	-24.46	0	0
	0,6	0	0	-2.01	-3.02	22.76	1.30
	2,6	92.72	86.92	0	0	1.98	5.91
C A S E IV	0,0	0	0	-17.94	-26.91	0	0
	0,6	0	0	-5.54	-8.32	17.63	3.58
	2,6	89.82	156.45	0	0	1.63	10.65
C A S E V	0,0	0	0	-19.03	-28.54	0	0
	0,6	0	0	-5.88	-8.82	11.88	3.99
	2,6	75.33	199.92	0	0	1.38	13.61
C A S E VI	0,0	0	0	-17.94	-26.91	0	0
	0,6	0	0	-5.54	-8.32	7.58	3.58
	2,6	57.95	255.99	0	0	1.08	15.38
C A S E VII	0,0	0	0	-16.31	-24.46	0	0
	0,6	0	0	-5.04	-7.56	5.2	3.25
	2,6	46.35	243.37	0	0	0.86	16.56
C A S E VIII	0,0	0	0	-14.68	-22.02	0	0
	0,6	0	0	-4.54	-6.8	3.70	2.92
	2,6	37.66	252.07	0	0	0.67	17.15
C A S E IX	0,0	0	0	13.59	-20.39	0	0
	0,6	0	0	-4.2	-6.3	2.53	2.71
	2,6	28.98	260.76	0	0	0.55	17.74
C A S E X	0,0	0	0	-12.5	-18.75	0	0
	0,6	0	0	-3.86	-5.79	1.96	2.49
	2,6	24.92	267.28	0	0	0.45	18.19

## DESIGN OF GRID

### TORSION

SPAN(M)	SHEAR		TORSION				Transverse RF		
	$\tau_c = 0.55 \text{ N/mm}^2$ $\tau_{cmax} = 2.5 \text{ N/mm}^2$	$\tau_v = 0.16 \text{ N/mm}^2$ $\tau_v < \tau_c$ ∴ Provide a min RF of 2 legged 8mm $\phi$ stirrups @ 300mm c/c	FACTORED	$V_e = V + 1.6 (T/b)$	$\tau_{ve} = V_e/bd$ $\tau_{cmax} = 2.5$ $\tau_c = 0.55$	$M_e = \frac{M_u + M_t}{M_t} = \frac{T_u(1+D/b)}{1.7}$		Ast	Side RF
CASE-I (Span-4m)			M = 26.09 S = 20.49 T = 1.95	34.06 KN	0.262 N/mm <sup>2</sup>	30.23 KN-m	Ast = 148mm <sup>2</sup> (Provide 2 bars of 10 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 6mm $\phi$ @ 190mm c/c
CASE- II (Span-8m)			M = 86.9 S = 34.14 T = 14.67	136.19 KN	1.05 N/mm <sup>2</sup>	118.04 KN-m	Ast = 678mm <sup>2</sup> (Provide 4 bars of 16 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 6mm $\phi$ @ 190mm c/c
CASE-III (Span-12m)			M = 139.08 S = 34.14 T = 36.69	289.37 KN	2.23 N/mm <sup>2</sup>	216.96 KN-m	Ast = 925mm <sup>2</sup> (Provide 4 bars of 18 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 6mm $\phi$ @ 150mm c/c
CASE-IV (Span-16m)			M = 234.67 S = 26.45 T = 40.37	307.28 KN	2.36 N/mm <sup>2</sup>	320.37 KN-m	Ast = 1863mm <sup>2</sup> (Provide 2 bars of 20 mm $\phi$ ) Asc = 973mm <sup>2</sup> (Provide 4 bars of 18 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 12mm $\phi$ @ 190mm c/c
CASE-V (Span-20m)			M = 299.88 S = 20.42 T = 42.81	318.23 KN	2.46 N/mm <sup>2</sup>	390.76 KN-m	Ast = 2264mm <sup>2</sup> (Provide 8 bars of 20 mm $\phi$ ) Asc = 1392mm <sup>2</sup> (Provide 6 bars of 18 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 12mm $\phi$ @ 190mm c/c

SPAN(M)	$\tau_c = 0.55 \text{ N/mm}^2$ $\tau_{cmax} = 2.5 \text{ N/mm}^2$	FACTORED	$V_e = V + 1.6$ (I/b)	$\tau_{ve} = V_e/bd$ $\tau_{cmax} = 2.5$ $\tau_c = 0.55$	$M_e = \frac{M_u + M_t}{2}$ $M_t = \frac{M_u(1+D/b)}{1.7}$	Ast	Side RF	Transverse RF
CASE-VI (Span-24m)	$\tau_v = 0.18 \text{ N/mm}^2$ $\tau_v < \tau_c$ ∴ Provide a min RF of 2 legged 8mm $\phi$ stirrups @ 300mm c/c	M = 384 S = 23.07 T = 40.37	303.9 KN	2.34 N/mm <sup>2</sup>	470 KN-m	Ast = 2664mm <sup>2</sup> (Provide 8 bars of 22 mm $\phi$ ) Asc = 1810mm <sup>2</sup> (Provide 6 bars of 20 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 10mm $\phi$ @ 130mm c/c
CASE-VII (Span-28m)	$\tau_v = 0.19 \text{ N/mm}^2$ $\tau_v < \tau_c$ ∴ Provide a min RF of 2 legged 8mm $\phi$ stirrups @ 300mm c/c	M = 386 S = 24.84 T = 37	282.23 KN	2.17 N/mm <sup>2</sup>	464.54 KN-m	Ast = 2517mm <sup>2</sup> (Provide 8 bars of 20 mm $\phi$ ) Asc = 1656mm <sup>2</sup> (Provide 6 bars of 20 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 10mm $\phi$ @ 150mm c/c
CASE-VIII (Span-32m)	$\tau_v = 0.2 \text{ N/mm}^2$ $\tau_v < \tau_c$ ∴ Provide a min RF of 2 legged 8mm $\phi$ stirrups @ 300mm c/c	M = 388.11 S = 25.73 T = 33.03	255.5 KN	1.97 N/mm <sup>2</sup>	458.22 KN-m	Ast = 2543mm <sup>2</sup> (Provide 8 bars of 20 mm $\phi$ ) Asc = 1684mm <sup>2</sup> (Provide 6 bars of 20 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 8mm $\phi$ @ 110mm c/c
CASE-IX (Span-36m)	$\tau_v = 0.2 \text{ N/mm}^2$ $\tau_v < \tau_c$ ∴ Provide a min RF of 2 legged 8mm $\phi$ stirrups @ 300mm c/c	M = 391.14 S = 26.61 T = 30.59	239.41 KN	1.84 N/mm <sup>2</sup>	456.08 KN-m	Ast = 2583mm <sup>2</sup> (Provide 8 bars of 20 mm $\phi$ ) Asc = 1727mm <sup>2</sup> (Provide 6 bars of 20 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 10mm $\phi$ @ 190mm c/c
CASE-X (Span-40m)	$\tau_v = 0.2 \text{ N/mm}^2$ $\tau_v < \tau_c$ ∴ Provide a min RF of 2 legged 8mm $\phi$ stirrups @ 300mm c/c	M = 400.92 S = 27.29 T = 28.13	222.98 KN	1.72 N/mm <sup>2</sup>	460.63 KN-m	Ast = 2621mm <sup>2</sup> (Provide 6 bars of 25 mm $\phi$ ) Asc = 1766mm <sup>2</sup> (Provide 6 bars of 20 mm $\phi$ )	Provide 1 bar of 12 mm $\phi$ on each side	Provide 8mm $\phi$ @ 130mm c/c

## ESTIMATION OF GRIDS

SPANS	ESTIMATION OF CONCRETE				ESTIMATION OF STEEL				TOTAL COST (Rs.)
	SLAB (m <sup>3</sup> )	BEAM (m <sup>3</sup> )	TOTAL (m <sup>3</sup> )	RATE (Rs.)	SLAB (kg)	BEAM (kg)	TOTAL (kg)	RATE (Rs.)	
4m	7.2	7.62	14.82	48,165	236	258	494	7,915	56,080
8m	14.4	13.58	27.98	90,932	474	1,216	1,690	27,033	1,17,965
12m	21.6	19.54	41.14	1,33,708	711	2,844	3,555	56,883	1,50,590
16m	28.8	25.5	54.30	1,76,483	948	6,903	7,851	1,25,619	3,02,102
20m	36.0	31.46	67.46	2,19,258	1,185	10,913	12,098	1,93,562	4,12,820
24m	43.2	37.43	80.63	2,62,048	1,422	15,680	17,102	2,73,632	5,35,679
28m	50.4	43.39	93.79	3,04,818	1,659	16,560	18,219	2,91,504	5,96,322
32m	57.6	49.35	106.95	3,27,588	1,896	18,504	20,400	3,26,392	6,73,980
36m	64.8	55.31	120.11	3,90,358	2,133	24,030	26,163	4,18,610	8,08,967
40m	72.0	61.27	133.27	4,33,128	2,370	24,390	26,761	4,28,176	8,61,303

## DESIGN OF TRUSSES

For Span 4 m

a. Design of comp member (Top chord member)

$$\text{Axial force} = 5.2 \text{ KN}$$

$$\text{Assume S.R} = 100$$

$$F_y = 260 \text{ N/mm}^2$$

The permissible stress in axial comp =  $\sigma_{bc} = 82 \text{ N/mm}^2$

$$\text{Length of the member} = 1.12 \text{ m}$$

$$\text{Eff length} = 1.12 \text{ m}$$

$$\begin{aligned} \text{Area req for the comp member} &= \frac{5.2 \times 1000}{82} \\ &= 63.4 \text{ mm}^2 \end{aligned}$$

Since the area is very less value 80 provide min steel section as

2 ISA 50×50×6

Assume the legs are kept back to back and 6mm thick gusset plates

are used

$$\text{Sectional area provided} = 2 \times 568 = 1136 \text{ mm}^2$$

$$r_{\min} = 15.1$$

$$\text{S.R } l = \frac{l}{r_{\min}} = \frac{1.21 \times 1000}{15.1} = 74.17$$

For  $l = 74.17$  and  $f_y = 260 \text{ N/mm}^2$

$$\sigma_{bc} = 110 \text{ N/mm}^2$$

$\therefore$  Load carrying capacity =  $110 \times 1136 = 124.96 > 5 \text{ KN}$

Hence the design is safe.

b. Design of tension member (Bottom Chord member)

$$\text{Axial force} = 4.7 \text{ KN}$$

$$\text{Allowable stress in axial tension} = 0.6 \times 260 = 156 \text{ N/mm}^2$$

$$\text{Area req.} = \frac{4.7 \times 1000}{156} = 30.13 \text{ mm}^2$$

$$\text{Gross area} = 30.13 \times 1.4 = 42.18 \text{ mm}^2$$

From ISI hand book select 2ISA  $50 \times 50 \times 6$

The angles are provided on both sides of Gusset plates

Assume  $16 \text{ mm } \phi$  rivets are used in one row.

$$\text{Gross area of angle section} = 1136 \text{ mm}^2$$

$$\text{Net area provided} = 1136 - 2 \times 23.5 \times 6$$

$$= 854 \text{ mm}^2$$

$$\text{Allowable axial tension} = 156 \times 854 / 1000 = 133.22 \text{ KN}$$

$$133.22 \text{ KN} > 4.7 \text{ KN}$$

Hence the design is safe.

### c. Design of Intermediate member

Axias force = 7.5KN (comp member)

Assume SR = 100

$$F_y = 260\text{N/mm}^2$$

$$\therefore \sigma_{bc} = 82\text{N/mm}^2$$

Length of member = 0.5m

$$\text{Area req} = 7.5 \times 100 / 82 = 91.46 \text{ mm}^2$$

Select a single angle section of ISA 50×50×6

$$\text{Sectional area} = 568\text{mm}^2$$

$$r_{\min} = 15.1$$

$$\therefore l = 33.1$$

$$\therefore \sigma_{bc} = 151 \text{ N/mm}^2$$

$$\text{Load carrying capacity} = 151 \times 568 = 85.7 \text{ KN}$$

$$85.7\text{KN} > 7.5 \text{ KN}$$

Hence the design is safe

### Design of joint

Assume 16mm  $\phi$  rivets are used

$$\text{Strength of rivet in double stear} = 2 \times \frac{\pi}{4} \times 17.5^2 \times 100$$

$$= 48.1\text{KN}$$

$$\text{Strength of rivet in bearing} = 17.5 \times 6 \times 300$$

$$= 31.5\text{KN}$$

$$\text{Rivet value} = R = 31.5 \text{ KN}$$

NO of rivet req to connect top chord member

$$= 5.2 / 31.5 \cong 1$$

∴ All the connection are connected by using only one rivet.

### ESTIMATION OF TRUSS :

Span = 4m

Cost of AC sheet including laying charge = 132.5/m<sup>2</sup>

Cost of sections including fabrications = Rs. 8 Kg

Top chord member provided = 2ISA 50×50×6

Bottom chord member provided = 2ISA 50×50×6

Intermediate member provided = 2ISA 50×50×6

For purlins the section used is = ISMC 125 @ 12.7 Kg/m

Weight/m run of ISA 50×50×6 = 8 Kg

Weight/m run of 2ISA 50×50×6 = 4.5 Kg

Total length of top chord member = 4.48m

Therefore total Wt of steel = 4.48×8

$$= 35.8 \text{ Kg}$$

Total length of bottom chord member = 4m

Therefore total Wt of steel = 4×8

$$= 32 \text{ Kg}$$

$$\begin{aligned}
\text{Total length of intermediate member} &= 4.24\text{m} \\
\text{Therefore total Wt of steel} &= 4.24 \times 4.5 \\
&= 19.1\text{Kg} \\
\text{Total length of purlins} &= 75\text{m} \\
\text{Therefore Wt of steel} &= 75 \times 12.7 \\
&= 953\text{Kg} \\
\text{Total steel in single stress} &= 19.1+32+35.8 \\
&= 86.9 \text{ Kg} \\
\text{Steel req. for 6 Nos of truss} &= 6 \times 86.9 \\
&= 521.4 \text{ Kg} \\
\text{Total steel} &= 521.4+953 \\
&= 1474.4\text{Kg} \\
\text{Steel required for Guesset plate, reverting} \\
\text{and wastages, 5\% of total steel} &= 0.05 \times 1474.4 \\
&= 73.72 \text{ Kg} \\
\text{Total amount of steel required} &= 1548.12 \text{ Kg} \\
\text{Total Cost of steel} &= 1548.12 \times 8 \\
&= \text{Rs. } 12,385 \\
\text{Imckind area of AC sheef} &= 2.24 \times 15 \times 2 \\
&= 67.2\text{m}^2
\end{aligned}$$

$$\text{Cost required} = 67.2 \times 132.5$$

$$= \text{Rs.}8904$$

$$\text{Total Cost} = 8904 + 12385$$

$$= \text{Rs.}21,739$$

TRUSS

SPAN-- ---4METRE

INPUT DATA

8 13 3 2e10  
1 0 0  
2 1 .5  
3 2 1  
4 3 .5  
5 4 0  
6 3 0  
7 2 0  
8 1 0  
1 1 2 .002  
2 2 3 .002  
3 3 4 .002  
4 4 5 .002  
5 5 6 .002  
6 6 7 .002  
7 7 8 .002  
8 8 1 .002  
9 8 2 .002  
10 7 2 .002  
11 7 3 .002  
12 7 4 .002  
13 6 4 .002  
5  
1 0 140  
2 0 280  
3 0 280  
4 0 280  
5 0 140  
1 0  
2 0  
12 0

TRUSS

SPAN-----04METRE

MEMBER	NODE	AXIAL FORCE
1	1	-0.5217E+03
	2	0.5217E+03
2	2	-0.2087E+03
	3	0.2087E+03
3	3	-0.2087E+03
	4	0.2087E+03
4	4	0.3130E+03
	5	-0.3130E+03
5	5	-0.2800E+03
	6	0.2800E+03
6	6	-0.2800E+03
	7	0.2800E+03

7	7	0.4667E+03
	8	-0.4667E+03
8	8	0.4667E+03
	1	-0.4667E+03
9	8	-0.4883E-03
	2	0.4883E-03
10	7	-0.3130E+03
	2	0.3130E+03
11	7	-0.9333E+02
	3	0.9333E+02
12	7	0.5217E+03
	4	-0.5217E+03
13	6	-0.7467E+03
	4	0.7467E+03

	AXIAL FORCE (KN)	A <sub>REQ</sub> (mm <sup>2</sup> )	SELECTED SECTION	APPROVIDED (mm <sup>2</sup> )	L.C.C. (KN)	JOINT DESIGN	REMARKS
<u>For 4m Span</u> (i) Top Chord (i) Bottom Chord (ii) Intermediate	5.2	63	2ISA50x50x6	1136	125>5.2	No. of rivet = 1	S.R=100, fy=260 $\sigma_{bc}=82\text{N/mm}^2$ $\sigma_t = 156\text{ N/mm}^2$ Here $A_{pro} = A_{net}$ .
	4.7	30	2ISA50x50x6	854	133>4.7	No. of rivet =1	
	7.5	91	ISA 50x50x6	427	67>7.5	No. of rivet =1	
<u>For 8m SPAN</u> (i) Top Chord (i) Bottom Chord (ii) Intermediate	22	268	2ISA50x50x6	1136	125>22	No. of rivet = 1	S.R=100, fy=260 $\sigma_{bc}=82\text{N/mm}^2$ $\sigma_t = 156\text{ N/mm}^2$ Here $A_{pro} = A_{net}$ .
	19.6	126	2ISA50x50x6	854	133>19.6	No. of rivet =1	
	8.4	75	ISA 50x50x6	427	67>8.4	No. of rivet =1	
<u>FOR 12m SPAN</u> (i) Top Chord (ii) Bottom Chord (iii) Intermediate (Tension)	34	415	2ISA 50x50x6	1136	102>34	No. of rivet =2	S.R=100, fy=260 $\sigma_{bc}=82\text{N/mm}^2$ $\sigma_t = 156\text{ N/mm}^2$ Here $A_{pro} = A_{net}$ .
	30.4	273	2ISA 50x50x6	854	133>30.4	No. of rivet =2	
	13	117	ISA 50x50x6	427	67>13	No. of rivet =1	
<u>FOR 16m SPAN</u> (i) Top Chord (ii) Bottom Chord (iii) Intermediate (Tension)	66	805	2ISA 50x50x6	1136	117>66	No. of rivet =2	S.R=100, fy=260 $\sigma_{bc}=82\text{N/mm}^2$ $\sigma_t = 156\text{ N/mm}^2$ Here $A_{pro} = A_{net}$ .
	62	556	2ISA 50x50x6	854	133>62	No. of rivet =2	
	21	188	ISA 50x50x6	427	67>21	No. of rivets= 1	

	AXIAL FORCE (KN)	$A_{REQ}$ (mm <sup>2</sup> )	SELECTED SECTION	APPROVIDED (mm <sup>2</sup> )	L.C.C. (KN)	JOINT DESIGN	REMARKS
<u>FOR 20m SPAN</u> (i) Top Chord (ii) Bottom Chord (iii) Intermediate (Tension)	77 71 26	939 637 188	2ISA 50x50x6 2ISA 50x50x6 ISA 50x50x6	1136 854 427	125>77 133>71 67>26	No. of rivets = 3 No. of rivets = 3 No. of rivets = 2	S.R=100, fy=260 $\sigma_{bc}$ =82N/mm <sup>2</sup> $\sigma_t$ = 156 N/mm <sup>2</sup> Here $A_{pro} = A_{net}$ .
<u>FOR 24m SPAN</u> (i) Top Chord (ii) Bottom Chord (iii) Intermediate (Tension)	100 94 33	1220 844 320	2ISA 60x60x6 2ISA 50x50x6 ISA 60x60x6	1368 854 543	150>100 133>94 84>39	No. of rivets = 4 No. of rivets = 3 No. of rivets = 2	S.R=100, fy=260 $\sigma_{bc}$ =82N/mm <sup>2</sup> $\sigma_t$ = 156 N/mm <sup>2</sup> Here $A_{pro} = A_{net}$ .
<u>FOR 28m SPAN</u> (i) Top Chord (ii) Bottom Chord (iii) Intermediate (Tension)	119 112 39	1451 1005 350	2ISA 60x60x6 2ISA 50x50x6 ISA 60x60x6	1368 854 543	157>119 133>112 84>39	No. of rivets = 4 No. of rivets = 4 No. of rivets = 2	S.R=100, fy=260 $\sigma_{bc}$ =82N/mm <sup>2</sup> $\sigma_t$ = 156 N/mm <sup>2</sup> Here $A_{pro} = A_{net}$ .

	AXIAL FORCE (KN)	$A_{REQ}$ (mm <sup>2</sup> )	SELECTED SECTION	APPROVIDED (mm <sup>2</sup> )	L.C.C. (KN)	JOINT DESIGN	REMARKS
<u>FOR 32m SPAN</u>							
(i) Top Chord	144	1756	2ISA 65x65x8	1488	234>144	No. of rivets = 5	S.R=100, fy=260 $\sigma_{bc}=82N/mm^2$
(ii) Bottom Chord	136	1220	2ISA 60x60x6	1086	169>136	No. of rivets = 5	$\sigma_t = 156 N/mm^2$
(iii) Intermediate (Tension)	51	458	ISA 60x60x6	543	84>51	No. of rivets = 2	Here $A_{pro} = A_{net}$
<u>FOR 36m SPAN</u>							
(i) Top Chord	158	1927	2ISA 65x65x10	2400	276>158	No. of rivets = 6	S.R=100, fy=260 $\sigma_{bc}=82N/mm^2$
(ii) Bottom Chord	148	1328	2ISA 60x60x6	1086	169>148	No. of rivets = 5	$\sigma_t = 156 N/mm^2$
(iii) Intermediate (Tension)	52	467	ISA 60x60x6	543	84>52	No. of rivets = 2	Here $A_{pro} = A_{net}$
<u>FOR 40m SPAN</u>							
(i) Top Chord	175	2134	2ISA 65x65x10	2400	302>175	No. of rivets = 6	S.R=100, fy=260 $\sigma_{bc}=82N/mm^2$
(ii) Bottom Chord	164	1472	2ISA 60x60x10	1824	284>164	No. of rivets = 6	$\sigma_t = 156 N/mm^2$
(iii) Intermediate (Tension)	58	520	ISA 60x60x10	912	122>58	No. of rivets = 3	Here $A_{pro} = A_{net}$

**COMPARISON OF QUANDITIES**

SPAN (m)	FRAME		GRID		TRUSS STEEL
	CONC QTY (m <sup>3</sup> )	STEEL QTY (kg)	CON QTY (m <sup>3</sup> )	STEEL QTY (kg)	
4.	12.72	459	14.82	494	1548
8.	28.32	947	27.98	1690	3125
12.	44.64	1747	41.14	3555	3788
16.	68.16	6487	54.30	7851	5375
20.	88.20	7674	67.46	12098	7365
24.	115.20	11340	80.63	17102	8907
28.	144.48	19725	93.79	18219	11280
32.	176.64	20840	106.95	20400	14690
36.	205.20	31074	120.11	26163	17508
40.	235.20	34621	133.27	26761	28814

## CONCLUSION & DISCUSSION

From the study carried out in this project, comparison between three roofing systems i.e, framed structures, Grid structures & Trusses was done and the following results obtained.

1. A graph is plotted showing the costs of Truss, Grid and Frame roofing structures upto 40m spans. (Graph 1).
2. A table showing a comparative increase in percentage in cost of frame and Grid structures to that of trusses. (Table 1)
3. A graph plotted for truss from which an equation was derived which can be used to find the weight of trusses for any span. (Graph 2).

### CONCLUSION :

The following conclusions were made from the results available from this project.

1. The cost of truss remains less when compared to that of Grid and frame structures for any span.

2. When comparing with the costs of frame and Grid structures upto 16m span the cost of framed structure remains less and from 16m to 24m the cost of Grid is comparatively less. Any how the variation in costs between these two structures upto 25m is less.
3. From 24m upto 40m and above, the cost of Grid structures remains economical as the cost of frames increases rapidly.
4. The percentage of cost variation of frames when compared to Trusses varies from 124% upto 318% for spans upto 40m.
5. The percentage of cost variation of Grid structures when compared to trusses varies from 158% to 173% for spans upto 40m.
6. An expression is derived for calculating the weight of truss for the known value of span.

$$W=KL^2$$

$$K = 11 \text{ ( For span 0 to 12m)}$$

$$= 3.2 \text{ (For span 12 to 24m)}$$

$$= 2.2 \text{ (For span 24m to 40m)}$$

W = Weight of truss in "kg"

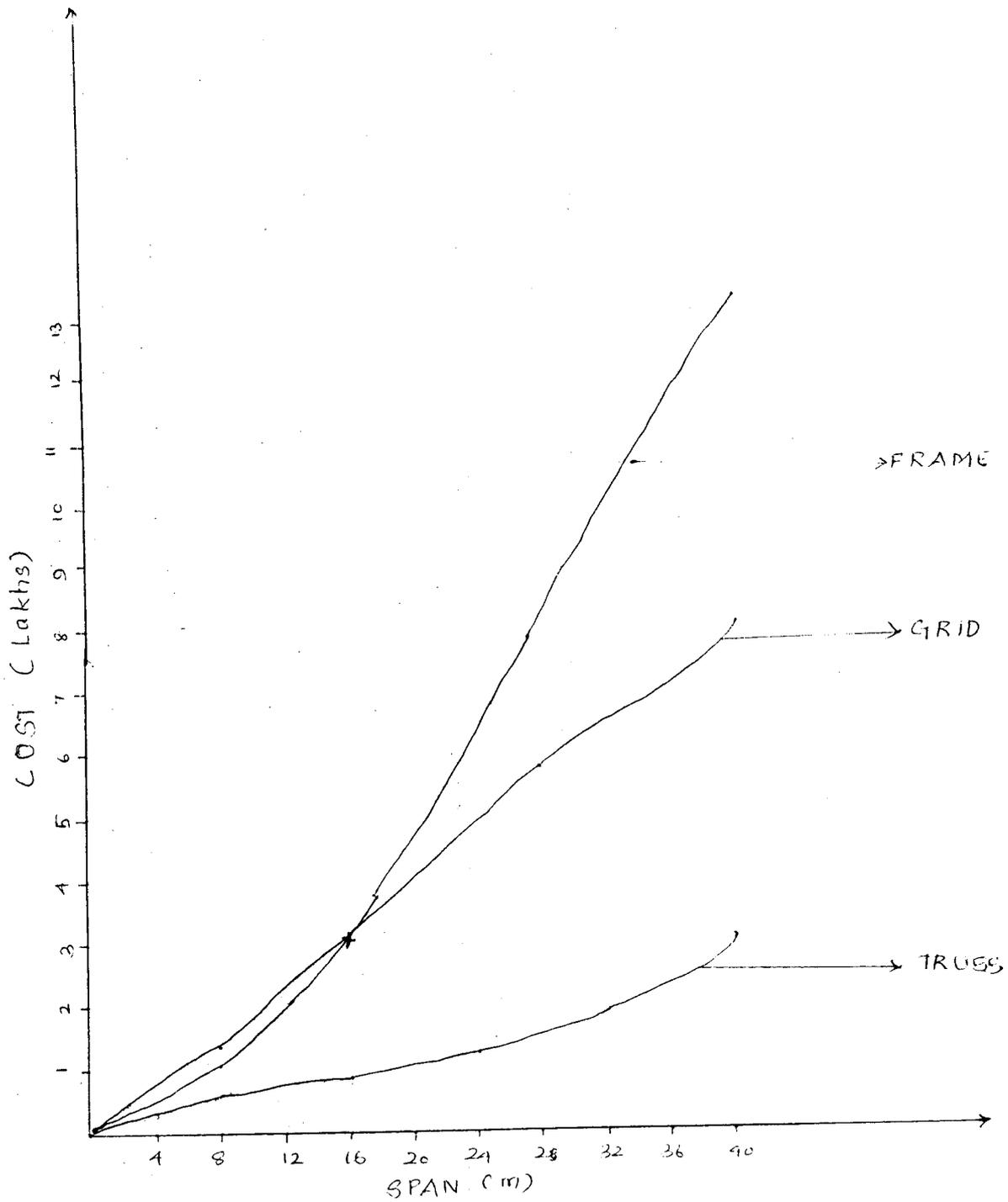
l = Span of truss in metre.

## DISCUSSION :

1. Since the cost of trusses seems to be less for any span, preference can be given to these structures where only economy is considered.
2. The variation of costs of frames are less up to 12m span and moderate up to 24m span, this is because at the depth of the frame is fixed depending upon the strength criteria and above 24m the depth of frames increases leading to uneconomy and this is caused because the depth of frames are depending upon the serviceability criteria.
3. The cost variation between frames and Grids doesn't have much variation right from 1m span up to 40m span apart from the spans up to 12m where the cost of frame is less. The rest seems to be in favour of the Grid structures.

Hence Grid structures can be preferred than framed structures as it has better appearance than both frames and also trusses.

# GRAPHICAL COMPARISON OF COST (Graph-1)



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